DISPLACEMENT-BASED DESIGN OF WOODFRAMED STRUCTURES WITH LINEAR VISCOUS FLUID DAMPERS

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ABSTRACT :

Activities over the last few years within the NEESWood project have focused on the development of a performance-based seismic design philosophy for woodframed structures. One approach to improving the performance of such structures is to utilize an advanced seismic protection system (e.g., a seismic isolation or damping system). Full-scale testing of an energy dissipation system within a woodframed building has been conducted by the authors as part of the NEESWood project. The testing involved the application of prefabricated modular damper walls to a two-story, 1,800 sq ft woodframed townhouse structure that was tested on the seismic shaking tables at the University at Buffalo NEES site. Based on what was learned from this testing, a new toggle-braced design for the modular damper walls was developed and is currently undergoing testing This paper presents a displacement-based design procedure for application of toggle-braced linear viscous dampers to multistory woodframed structures. The procedure was originally developed within the NEESWood project and in this study is further extended to include the application of a damping system. The procedure requires simple modal analysis and determination of equivalent stiffness based on the backbone response of the shearwalls. As an illustrative example, the design of the benchmark test structure retrofitted with a toggle-braced damping system is presented. The validity of the proposed displacement-based procedure is confirmed using results from nonlinear dynamic response-history analyses.

KEYWORDS: Energy dissipation, dampers, woodframed buildings, performance-based seismic design

1. INTRODUCTION

Although woodframed construction in the U.S. has generally been considered to perform well during earthquakes, the 1994 Northridge Earthquake (Moment Magnitude = 6.8) clearly demonstrated the seismic vulnerability of such construction. As such, the NEESWood project seeks to develop a performance-based seismic design (PBSD) philosophy that will provide the necessary mechanisms to safely increase the height of woodframed structures in active seismic zones of the U.S. as well as mitigating damage to low-rise woodframed structures (van de Lindt et al. 2006). The testing phase of the NEESWood Project began with seismic shaking table testing of a full-scale, two-story, woodframed townhouse building (the "benchmark" test structure) (Filiatrault et al. 2007). The results of the tests are being used to develop the PBSD philosophy. This PBSD philosophy will then be used to design a mid-rise (six-story) woodframed apartment building which will be tested using three-dimensional earthquake ground motions at the E-Defense shaking table facility in Miki City, Japan.

Phase 2 of the NEESWood benchmark structure test program involved implementation and evaluation of a seismic damping system (Shinde et al. 2007). Due to a number of factors, including the inherent flexibility in the connections of wood framing systems, engagement of the dampers was limited during these tests and thus the full effectiveness of the dampers was not realized. Based on what was learned from this testing, a new design for the modular damper walls with a toggle brace configuration has been developed and is currently undergoing testing. The objective of the study described herein is to modify a displacement-based seismic design procedure (that has been developed within the NEESWood project) to accommodate the inclusion of toggle-braced seismic dampers in woodframed buildings. The design procedure is presented using performance levels/metrics being articulated as part of the NEESWood project and the procedure is validated using results from nonlinear response-history analyses.

2. OVERVIEW OF DIRECT DISPLACEMENT-BASED SEISMIC DESIGN

Filiatrault and Folz (2002) presented one possible displacement-based design procedure for wood structures, with the procedure originating from one developed by Priestley (1998). This design procedure has been applied to a two-story woodframed building (Filiatrault et. al. 2006). A new displacement-based design procedure has been developed by Pang and Rosowsky (2007) within the NEESWood project. The procedure requires simple modal analysis and determination of equivalent stiffness based on the backbone response of the shearwalls that are being considered for the design. In the design process, the acceleration response spectrum is converted into a set of inter-story drift spectra which are used to determine the minimum stiffness required for each story such that the inter-story drift is limited to specified values.

The design of seismic damping systems within the context of PBSD procedures has been discussed within the literature. Kim et al (2003) utilized the capacity spectrum method for seismic retrofit of existing structures using viscous dampers. Kim and Choi (2006) presented displacement-based design with dampers for seismic retrofit of framed structures. In this study, the PBSD procedure developed by Pang and Rosowsky (2007), which is a direct displacement-based design (DDD) procedure, has been modified for designing a damping system for seismic retrofit of an existing woodframed structure. The modification incorporates the traditional procedure of determining the size and location of energy dissipation devices (dampers) based on FEMA 356 guidelines (ASCE 2000).

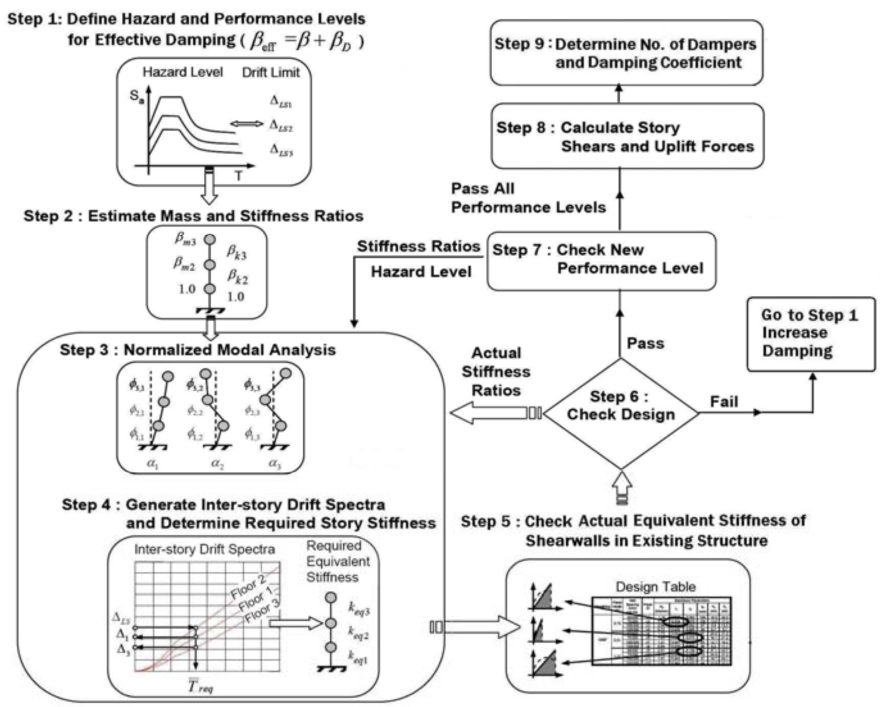


Figure 1 Flowchart for Designing Dampers for Seismic Retrofit of Multi-Story Buildings using DDD Procedure (Adapted from Pang and Rosowsky 2007)

A flowchart that summarizes the key steps in designing a damping system for seismic retrofit applications is shown in Figure 1. A detailed description of many of these key steps, including normalized modal analysis, construction of inter-story drift spectra, development of wood shearwall design tables to determine actual equivalent stiffness, and computation of story shear and uplift forces, can be found in Pang and Rosowsky (2007).

3. DESIGN EXAMPLE

3.1.Building Configuration

The structure selected for this DDD example is the benchmark test structure that was tested on the seismic shaking tables at the University at Buffalo NEES site in 2006 (see Figure 2). This building is assumed to have been built as a "production house" in either the 1980's or 1990's, located in either Northern or Southern California. The structure is a two-story townhouse building (approximately 1600 ft^2) with an attached two-car garage. Effective seismic weights of 55.5 kips and

23.5 kips are assigned to the first and second story, respectively. A detailed description of the test structure, experimental test setup, instrumentation, and shake table test program (test phases) is provided by Filiatrault et al. (2007). The proposed DDD procedure will be presented within the context of a retrofit application wherein seismic dampers (linear viscous fluid dampers) are incorporated within the benchmark test structure. In this study, analysis and design are performed only in the weaker direction (transverse) and for the Phase 4 test structure which includes gypsum wallboard installed on all structural and partition walls and ceilings. Note that the Phase 5 benchmark test structure (shown in Figure 2) includes exterior finish material but was not considered for the analysis herein due to lack of readily available data on the parameters that define the hysteretic behavior of the fully finished shear walls.

Figure 2 Phase 5 Benchmark Test Structure

3.2. Design Process

3.2.1. Target Performance Levels and Seismic Hazard Levels (Step 1 in Figure 1)

Table 1 shows performance levels and the corresponding drift limits for woodframed structural assemblies (modified from FEMA 356). The aforementioned benchmark structure tests indicated that the collapse prevention limit state for a two-story building is not 3%, as specified in FEMA 356, but rather is significantly higher. Thus, the collapse prevention drift limit was taken as 5% in this study. A study investigating the original FEMA limit states for woodframed buildings can be found in van de Lindt and Liu (2007).

Table 1 Structural Performance Levels for Woodframed Structural Assemblies

	Structural Performance Levels		
	Collapse Prevention (CP)	Life Safety (LS)	Immediate Occupancy (IO)
Drift	5% transient	2% transient	1% transient
Limits	Or permanent	1% permanent	0.25% permanent

The design acceleration response spectra along with shape defining parameters (based on the USGS 2002 hazard data and FEMA 356 procedure) are shown in Figure 3 with the assumption that the benchmark structure is located at City Hall in Los Angeles, California. According to Section 1.6.1.5.3 of FEMA 356 (ASCE 2000), for structures rehabilitated using seismic isolation or supplemental energy dissipation, the design acceleration response spectrum should be constructed using an equivalent effective viscous damping ratio, β_{eff}, which is computed using the procedure specified in Chapter 9 wherein the short-period and one-second period damping coefficients B_S and B_1 defined in Table 1-6 (see Table 2) are utilized. Herein, since nonlinear hysteretic damping is explicitly considered in the analysis, the inherent viscous damping of the structure is taken as 5% in the fundamental mode.

3.2.2. Normalized Modal Analysis and Construction of Inter-story Drift Spectra (Step 3 and 4 in Figure 1)

The mass ratios, β_m, (relative to the first story) are 1.0 and 0.42 for the first and second stories, respectively. The target stiffness ratio β_k must be estimated based on engineering judgment/experience. Initial values of β_k can be estimated based on the total full-height shearwall length in the direction considered and for each story. The total shearwall length for the first and second story in the weaker direction of the benchmark structure is about 52 ft and 49 ft, respectively. Based on the shearwall length, an initial estimate of β_k = 0.94 can be assumed for the second story. However, since the inertial mass driving the second story is relatively small compared to the first story, an initial value of β_k = 2.5 for the second story was assumed. After the mass and stiffness ratios have been determined, a normalized modal analysis is performed to compute frequency parameters α_n and mode shapes ϕ_n along the transverse direction of the structure. Knowing these parameters, the inter-story drift factors γ_{jn} are computed using:

$$\gamma_{jn} = \Gamma_n \left(\phi_{jn} - \phi_{j-1,n} \right) \tag{3.1}$$

where Γ_n is the modal participation factor corresponding to the n-th mode. The design inter-story drift spectra are then generated from the design acceleration response spectrum using:

$$\Delta_j \left(\overline{T} \right) = \frac{1}{H_j} \sqrt{ \sum_n \left[\gamma_{jn} \left(\frac{\overline{T}}{2\pi\alpha_n} \right)^2 S_a \left(\frac{\overline{T}}{\alpha_n} \right) \right]^2 } \tag{3.2}$$

where $\Delta_j \left(\overline{T} \right)$ is the inter-story drift for the j-th story (with contributions from all modes), H_j is the height of the j-th story, and $\overline{T}$ is the normalized first-story period. An example of the inter-story drift spectra for the structure without dampers and for a 2%/50 year hazard level, along with the parameters used for its generation, is shown in Figure 4. To meet the 5% drift limit associated with the CP performance level (see Table 1), the maximum normalized first-story period, $\overline{T}_{req}$, is 0.43 sec. At the same hazard level, the second story will experience only 0.64% drift. Knowing the required first-story period $\overline{T}_{req}$, the required equivalent stiffness of each story is then determined using:

$$\left(k_{eq} \right)_j = \left(\frac{2\pi}{\overline{T}_{req}} \right)^2 m\beta_{kj} \tag{3.3}$$

where m is the inertial mass assigned to the first story and β_{kj} are the stiffness ratios (relative to the first story).

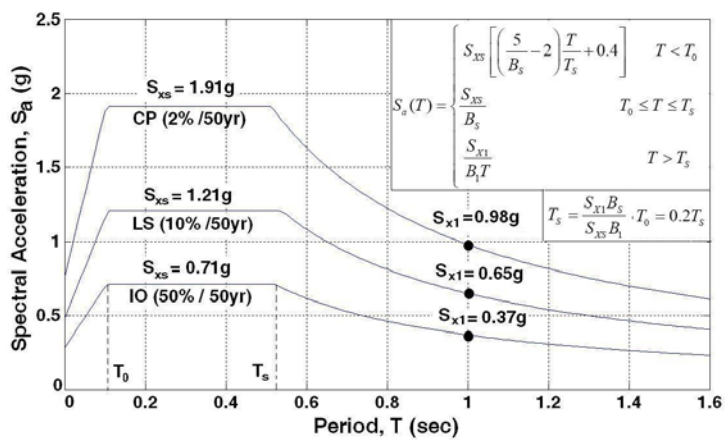


Figure 3 5%-damped Design Acceleration Response Spectra for Los Angeles, California

Table 2 Damping Coefficients B_S and B_1 from Table 1-6 in FEMA 356 (ASCE 2000)

Effective Viscous Damping β_{eff} (% of critical damping)*	≤ 2	5	10	20	30	40	≥ 50
B_S	0.8	1.0	1.3	1.8	2.3	2.7	3.0
B_1	0.8	1.0	1.2	1.5	1.7	1.9	2.0

*Linear Interpolation should be used for values other than those given

3.2.3. Verification of Design Using Equivalent Stiffness Ratios (Step 6 in Figure 1)

A shearwall design table for 8 ft high walls with studs spaced at 16 in on-center and OSB attached using 8d common nails is shown in Table 3. The table contains the values of parameters that define the backbone curve for walls constructed with various nailing patterns and panel widths. The CASHEW program (Folz and Filiatrault 2001), along with available shearwall test data from the benchmark test, was used to generate the design table.

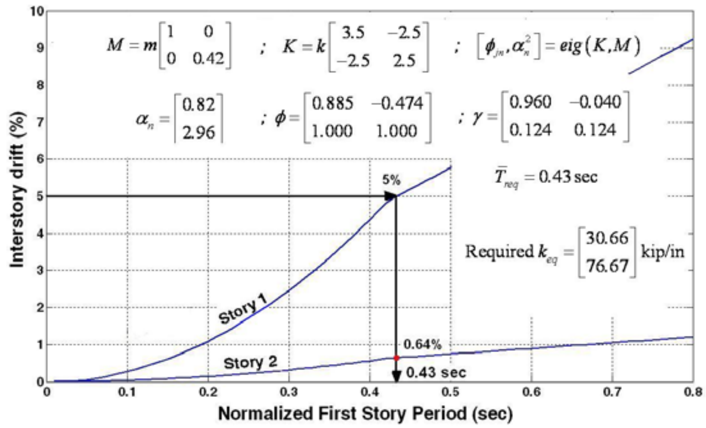


Figure 4 Inter-story Drift Spectra for 2%/50 year Hazard Level

Table 3 Shearwall Design Table for 8ft High Walls with Studs Spaced at 16 in On-Center and OSB Attached Using 8d Common Nails

Sheathing	Panel Width (ft)	Nail Spacing (in)	Panel ID	Backbone Parameters K_0 (kips/in)	r_1	r_2	δ_u (in)	F_0 (kips)	F_u (kips)	Equivalent Stiffness, K_{eq} (kips/in) at Target Drift, Drift (%) 0.5	1.0	2.0	5.0
OSB and GWB	2.5	4/12	s1	6.45	0.017	-0.053	3.74	2.63	3.01	4.57	3.43	2.28	1.08
		6/12	s2	5.25	0.011	-0.047	3.58	1.91	2.11	3.60	2.63	1.65	0.80
	3.0	4/12	s3	8.68	0.018	-0.054	3.19	3.28	3.78	6.00	4.45	2.91	1.37
		6/12	s4	7.20	0.012	-0.044	3.03	2.40	2.65	4.74	3.43	2.17	0.97
	4.0	3/12	s5	15.07	0.021	-0.084	2.76	6.27	7.12	10.79	8.22	5.42	2.40
		4/12	s6	13.70	0.017	-0.069	2.64	4.94	5.57	9.31	6.91	4.40	1.88
		6/12	s7	11.82	0.012	-0.053	2.48	3.66	4.00	7.59	5.37	3.31	1.37

Table 4 summarizes the displacement-based design of the benchmark structure without dampers in the transverse direction and for multiple performance levels. The actual equivalent stiffness provided by the shearwalls at each story varies from the required equivalent stiffness. Another normalized modal analysis is performed (using the actual values of β_k) to determine new story drift estimates and required equivalent stiffness. Finally, the ratio of actual and required equivalent stiffness is calculated to give an indication as to whether the design meets the given performance requirements. For example, as shown in Table 4, the benchmark structure without dampers does not satisfy CP and LS design limits for 2%/50 year and 10%/50 year hazard levels, respectively.

Table 4 Displacement-Based Design of Benchmark Structure Without Dampers in Transverse Direction for Multiple Performance Levels

Hazard Level	Perf. Level	Drift Limit (%)	Story	Initial β_k	Drift (%)	Required K_{eq} (kips/in)	Rounded Drift (%)	Actual K_{eq} (kips/in)	Actual β_k	Drift (%)	Required K_{eq} (kips/in)	K_{eq} Actual/ Required
2%/ 50yr	CP	5.00	1	1.00	5.00	30.66	5.00	21.02	1.00	5.00	30.66	0.69
			2	2.50	0.64	76.67	0.50	88.84	4.22	0.37	129.38	0.69
10%/ 50yr	LS	2.00	1	1.00	2.00	49.04	2.00	47.28	1.00	2.00	49.04	0.96
			2	4.22	0.22	206.98	0.50	88.84	1.88	0.35	92.19	0.96
50%/ 50yr	IO	1.00	1	1.00	1.00	59.00	1.00	73.71	1.00	1.00	59.00	1.25
			2	1.88	0.18	110.92	0.50	88.84	1.20	0.30	70.80	1.25

3.3. Displacement-Based Design of Benchmark Structure With Dampers

For the benchmark structure with dampers, new performance levels were defined as follows: LS performance for 2%/ 50 year hazard level and IO performance for 10%/ 50 year hazard level. Table 5 shows the ratio of actual and required equivalent stiffness for three different effective damping ratios (15%, 20% and 25%) as obtained from DDD analysis. Note that, for a given drift level, these ratios make sense in that the required equivalent stiffness reduces with increased effective damping (and thus the equivalent stiffness ratio reduces). Furthermore, these results indicate that the LS and IO performance levels can be achieved for the 2%/50 yr and 10%/50 yr hazard levels, respectively. However, a more refined nonlinear dynamic response-history analysis could be performed to confirm the results although it is expected that such analyses would yield similar inter-story drift demand results. The advantage of the displacement-based design is that it is much simpler than nonlinear dynamic response-history analysis while providing a reasonable prediction of structural performance for a given hazard level. For example, the results in Table 5 indicate that, for a 2%/50 yr hazard level, the IO performance level can not be achieved without increasing the effective damping to some value beyond 25%. Further, for a 10%/50 yr hazard level, the IO performance level can be achieved with an effective damping ratio of 15%. Based on the results in Table 5, an effective damping ratio of 20% was selected as an optimal design to achieve the desired performance for the given hazard levels. Fine tuning of the design could be performed by conducting nonlinear dynamic response-history analyses.

Table 5 Displacement-Based Design of Benchmark Structure With Dampers

Hazard Level	**Performance Level**	**Drift Limit (%)**	K_{eq} **Actual/** K_{eq} **Required**					
			β_{eff} = 15%		β_{eff} = 20%		β_{eff} = 25%	
			Story 1	Story 2	Story 1	Story 2	Story 1	Story 2
2%/50 yr	CP	5.00	1.25	1.25	1.51	1.52	1.76	1.76
	LS	2.00	0.96	0.96	1.11	1.11	1.27	1.27
	IO	1.00	0.75	0.75	0.87	0.88	0.98	0.99
10%/50 yr	LS	2.00	1.51	1.51	1.72	1.73	2.00	2.01
	IO	1.00	1.17	1.17	1.33	1.34	1.55	1.55

3.3.1 Determining the Number of Dampers and Damping Coefficient Values (Step 9 in Figure 1)

As mentioned previously, according to Section 1.6.1.5.3 of FEMA 356 (ASCE 2000), for structures retrofitted using seismic isolation or supplemental energy dissipation, an effective viscous damping ratio, β_{eff}, shall be calculated using the procedure specified in Chapter 9 and utilizing damping coefficients B_S and B_1 as defined in Table 1-6 (see Table 2). Thus, to determine the number of dampers and associated damping coefficient values, Equation 9-30 can be used:

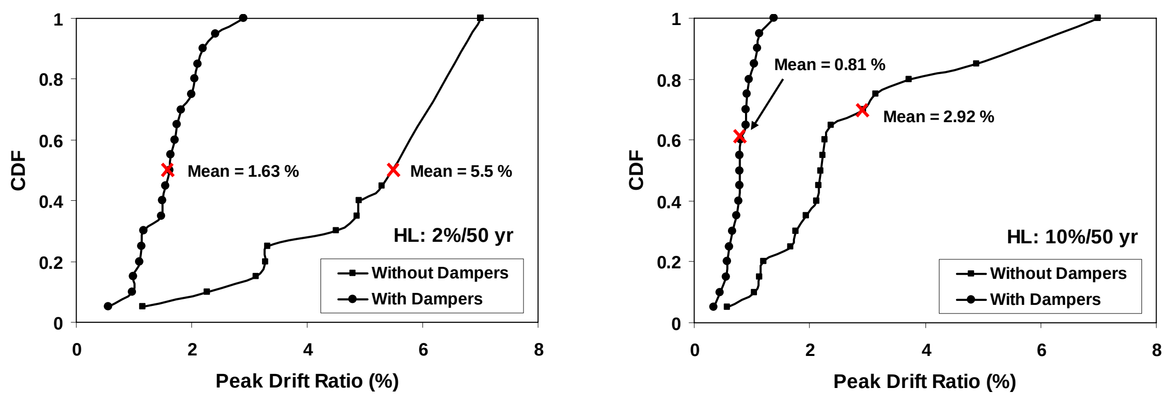
$$\beta_{eff} = \beta + \beta_D = \beta + \frac{T\sum_j C_j \cos^2\theta_j \phi_{rj}^2}{4\pi \sum_i \left(\frac{w_i}{g}\right)\phi_i^2} \tag{3.4}$$

where β is the inherent structural damping (taken as 5%), β_D is the supplemental damping, T is the fundamental period of the retrofitted building, C_j is the damping coefficient of damper j, θ_j is the angle of inclination of damper *j* with respect to the horizontal, ϕ_{rj} is the first mode displacement between the ends of damper *j* in the horizontal direction, ϕ_i is the first mode displacement at floor level *i*, and w_i is the seismic weight assigned to floor level *i*. Having selected the number of dampers, the Eq. 3.4 can be used to determine the required damping coefficient for each damper. For the optimal design presented above for the benchmark structure, the effective damping is 20% and thus the supplemental damping is equal to 15%. Note that in Eq. 3.4, $\cos\theta_j$ represents the magnification factor associated with the orientation of the damper with respect to the horizontal. An alternate damper configuration employs a toggle-brace wherein the damper displacement is amplified in accordance with the geometry of the toggle-brace assembly (Constantinou et al. 2006). In this study, a toggle-braced damper assembly is assumed to be used in the retrofit of the benchmark structure and the magnification factor is taken as 1.65 to correspond with an assembly that is currently undergoing testing. The configuration for lateral story drifts needs to be assumed to determine the number of dampers and their damping coefficient. In this case, it was assumed that the maximum story drifts are proportional to the fundamental mode shape. Alternatively, the maximum story drifts can be estimated using results from a pushover analysis. The frequencies and mode shapes were computed using the mass and initial stiffness of the benchmark structure. Assuming 4 dampers, each having the same damping coefficient, results in a damping coefficient value of 0.182 kip-sec/in. Note that a damping coefficient value of 0.225 kip-sec/in (20% higher than calculated value) was used for nonlinear dynamic response-history analysis to match the value associated with a toggle-brace damper assembly that is currently undergoing testing. This is considered to be reasonable since Eq. 3.4 is an approximate expression. It is important to realize that the analysis presented herein assumes full engagement of the dampers. Ongoing tests are exploring the efficiency of damping devices within woodframed shear walls.

4. DESIGN APPRAISAL

The optimal design of the damping system was determined based on a DDD procedure and, as a verification step in the PBSD procedure, earthquake simulations using a suite of twenty earthquake ground motions (Krawinkler et al. 2000) were performed. Similar to the work by Krawinkler et al. (2000), these earthquake ground motions were scaled such that their mean 5%-damped spectral values over a period range from 0.1 to 0.6 seconds is equal to 1.1g and 1.7g corresponding to the flat region of the 1997 Uniform Building Code design response spectrum for Los Angeles, CA for a 10%/50 yr and 2%/50 yr hazard level, respectively (ICBO 1997). The approach used in the present study is consistent with ground motion scaling described in the FEMA 302 (BSSC 1997) and FEMA 356 (ASCE 2000).

The simulation results (with and without dampers) for the 20 motions were plotted as cumulative distribution functions (CDF) (see Figure 5). Note that the first story drift ratio (which corresponds to the peak drift ratio of the structure) for the no damper case exceeds the 5% drift limit (mean value = 5.5%) corresponding to the CP performance level (thus indicating collapse or incipient collapse of the structure) for 7 of the 20 ground motions represented in Figure 5a which is consistent with the results from the DDD analysis (see Table 4). Note that a drift limit of 7%, a reasonable value for defining impending instability of the structure from an analysis point of view, was imposed on the calculation of mean drift. For the case with dampers, according to FEMA 302 it would be acceptable, if this were a force-based design, to design to the mean value of the CDF. Thus, for the selected damper design (β_{eff} = 20%), it is apparent that the LS performance level can be achieved for a 2%/50 yr hazard level as evidenced by the very low exceedance probability (mean value = 1.63%). This result is consistent with that from the DDD analysis (see Table 5). Therefore, it can be concluded that the design of the building with dampers meets (or exceeds) the design objective for life safety for a 2%/50 yr hazard level.



(a) **(b)**

Figure 5 Peak Drift Distributions from Nonlinear Dynamic Analysis of Benchmark Structure With and Without Dampers and for (a) 2%/ 50 yr and (b) 10%/ 50 yr Hazard Levels.

The structure was also analyzed for the 10%/50 yr hazard level. For the case without dampers, Figure 5b indicates that the structure is predicted to not achieve the LS performance level by a relatively small margin (mean value = 2.92%). This result is consistent with the DDD analysis (see Table 4). Note that the expected drift and the ratios of actual-to-required equivalent stiffness are inversely proportional and thus the drift demand of the designed structure can be estimated at each performance level by taking the ratio of the drift limit to the stiffness ratio (Pang and Rosowsky 2007). In this case, the drift demand should be approximately to 2.08% (i.e., 2/0.96 per data from Table 4) which is considerably different from the mean value of 2.92%. Note that the drift level for 3 of the 20 ground motions represented in Figure 5b exceeds the aforementioned 7% limit whereas the drift for 13 of the 20 motions is less than 2.5%. Hence, the mean value of 2.92% is not a good representation of the majority of drift values. For the optimal damper design case, the mean peak drift ratio was 0.81% which is less than the 1% drift limit required at the 10%/50 yr hazard level to meet the IO performance level. This is consistent with the DDD analysis (see Table 5). In this case, the drift demand should be equal to 0.75% (i.e., 1/1.33 per data from Table 5) which is very close to the mean value. Note that in the case of 2% /50 yr hazard level, the drift demand is also predicted well by this approximate method (i.e., the mean value of 1.63% is approximately equal to 2/1.11 = 1.80% per data from Table 5). Note that this approximate method of estimating drift demand was not considered to be applicable to the case without dampers for the 2%/50 yr hazard level (mean value = 5.5%) since 7 of the 20 ground motions produced drift levels that exceeded the 7% limit and thus the mean value is not a realistic value. Interestingly the estimated drift demand (i.e., 5/0.69

= 7.24%) is close to the 7% limit.

The reasonably good agreement between the design inter-story drifts (based on DDD analysis) and the mean peak drifts obtained from response-history analyses serves to validate the proposed modifications to the DDD procedure for inclusion of linear viscous dampers in seismic retrofit applications. Therefore, it can be concluded that the optimal design of the supplemental damping system used for retrofit of the benchmark structure satisfies the specified performance objectives.

5. CONCLUDING REMARKS

A direct displacement-based PBSD procedure for design of multi-story woodframed buildings with supplemental linear viscous dampers has been presented. The proposed method can be applied to the retrofit of existing structures or to the design of new structures. The method was applied for retrofit of the NEESWood Benchmark Structure with the final design being evaluated using nonlinear response-history analyses. The results demonstrated that the design inter-story drifts based on selected performance levels were in reasonable agreement with mean peak inter-story drifts obtained from the nonlinear response-history analyses, thus validating the proposed DDD procedure.

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